EUROCODE FOR HIGH STRENGTH STEEL AND APPLICATIONS IN CONSTRUCTION

Bernt Johansson – Luleå University of Technology, Sweden
Peter Collin.- Luleå University of Technology, Sweden

ABSTRACT

The construction industry is lagging behind other industries concerning use of high strength steel. One obstacle is that Eurocode 3 limits its scope to S460. A working group within CEN/TC250/SC3 has recently finished a Part 1-12 of Eurocode 3 that extends its scope to S700. Part 1-12 gives a few changes and some additional rules to the already existing parts of Eurocode 3 in order to make them applicable to steel grades up to S700. Basically, elastic analysis should be used but plastic resistance can still be utilised. For connections there are some restrictions for design methods utilising large plastic deformations. For welded connections undermatched electrodes will be allowed. The content of those new rules will be presented. The working group has also taken on to write a state of art report on the use of high performance steel in Europe, USA and Japan. It will be published as an IABSE Structural Engineering Document. It is clear that Europe is lagging behind in spite of the fact that several applications in the construction sector are cost effective. Examples of applications in Europe will be presented and commented. Some times it is possible just to change the material to a higher grade but in many cases it is necessary to change the conceptual design in order to make full use of the higher strength. High strength steel usually comes as plates and sheets, which means that profiles have to be built up. Cold forming is an attractive fabrication method for thin and medium thick plates or sheets. It makes possible to produce efficient profiles in a cost effective way compared to welding the profiles from flat plates. Examples of such profiles will be shown.

KEYWORDS

High strength steel, design, ductility, buckling, economy, applications, hybrid girder.

INTRODUCTION

Over the past decades, high strength steel (HSS) has gained ground in the market of steel structures. The steel grade S355, considered to be a high strength steel 20 years ago, is now the predominant grade for hot rolled plates and H-sections in the US and Sweden. These products are today available also in the grades S420 and S460 as TM-steel (thermo-mechanically rolled). Plates of QT-steel (quenched and tempered) are commercially available in grades up to S1100 although not yet standardised above S960. The price increase is usually less than the increase of the yield strength and it is therefore always economical to use HSS in structures where the strength can be fully utilised. Considering also the fabrication costs it is even more favourable to use HSS. TM-steels and QT-steels with low alloy content are easy to weld also in high grades.

Reduced cost is not the only reason for using HSS. In some applications the reduction of the weight is an important advantage because the payload can be increased or the running

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expenses decrease. This is the case for vehicles and for mobile cranes and in these applications HSS is already widespread. An increasingly important concern is the environment and more specifically the use of energy and raw material. By using HSS less resources are spent for fulfilling a given function. Although steel is easy to recycle (50 % of the steel is recycled) there is a need for virgin material and fairly large amounts of energy are used also for the recycled steel. Environmental concern is therefore a valid reason for using HSS.

For each application and type of structural solution there is an upper limit for the strength that can be utilised. Other requirements, like deflection limitations or fatigue may govern or instability phenomena may reduce the advantage of HSS. The conclusion from this fact is that new structural solutions have to be developed in order to increase this limit.

1. EN 1993 PART 1:12

1.1 GENERAL

CEN/TC250/SC3 appointed a working group named WG690 with the task to develop rules (additions and changes) to make EN 1993 applicable to high strength steel. The group was chaired by the senior author of this paper and it was established on his initiative for reasons given in the introduction. The group has some 25 members including steel makers and researchers and it met seven times before the job was finished. The main result was a draft prEN 1993-1-12 with subtitle "Additional rules for the extension of EN 1993 up to steel grades S700" [1]. The draft was approved by SC3 in 2004 and the final edited version has been sent to CEN.

The basis of design rules is test results and although there is quite a lot available on HSS there are gaps compared to normal strength steel. The general behaviour of members is adequately covered but for some types of connections there are no test results. In case that there are reasons to think that a certain design procedure would not work for HSS it has been exempted.

1.2 MATERIALS

EN 1993-1-12 covers steels according to EN 10025-6 and EN 10149-2. The former deals with quenched and tempered steels delivered as flat plates and those included in 3-1-12 range from S500 to S690. The material standard goes up to S960 but such high strength has presently little applicability in construction. Test results for grades above S690 are limited and it would have been hard to get higher grades accepted for general use.

EN 10149-2 deals with TM steels for cold forming and the grades included in EN 1993-1-12 range from S500 to S700. These steels are usually delivered in coils and the thickness is limited to 16 mm. They can be used for roll forming of profiles and one common application in heavy steel structures is trapezoidal stiffeners. The delivery conditions in EN 10149 do not include a mandatory test of impact energy. It is therefore stated in EN 199 3-1-12 that impact energy tests should be specified as an option with the minimum energy 40 J at -20 ^{o}C.

1.3 TOUGHNESS AND DUCTILITY

EN 1993-1-1 requires that brittle fracture should be avoided by choosing a sufficiently tough material. Detailed rules are given in EN 1993-1-10 in which the material toughness is measured by its impact energy. This property is not ideal for fracture mechanical purpose but it is used because it is routinely measured by the steel makers. The method used in EN 1993-1-10 for deriving maximum material thicknesses has been verified for steel grades well above S690 and already the first version of it included S690. In EN 1993-1-12 some missing grades from EN 10025-6 are treated and also steels according to EN 10149-2.

Ductility of steel structures is a less well defined property. EN 1993-1-1 states some requirements on the mechanical properties of the steel but no general requirements on ductility of the structure. For special cases, like earthquake resistance and plastic hinges, so called capacity design is required. This requirement means that the plastic resistance should be maintained under sufficient plastic deformation. For steel structures in general there is nothing that prevents designs that lead to brittle structures. The reason is probably economic as it would add substantial costs if all joints had to be designed for capacity. The code relies here on the common sense of the designer without going into details. The same trust should be shown to designers using HSS.

The requirements on material ductility are stated in 3.2.2(1) of EN 1993-1-1 with recommended values as follows:

$f_u / f_y \geq 1,10$

Elongation at failure not less than 15%

$\varepsilon_u \geq 15 f_y / E$

There are no systematic studies behind the numerical values in these limitations and they can be seen as a description of the properties of the steels included in EN 1993-1-1. In the ENV 1993-1-1 similar requirements were stated as conditions for the use of plastic analysis but here they have been applied to all steel structures. The justification for making the requirements general is that they are needed also for certain design rules for connections. As some of the steels included in EN 1993-1-12 do not fulfil the requirements it was important for the WG690 to analyse which restrictions were needed in order to relax the requirements. In EN 1993-1-12 the two first requirements are relaxed to:

$f_u / f_y \geq 1,05$

Elongation at failure not less than 10%

The restrictions on structures made of steels fulfilling those relaxed requirements are mainly that plastic analysis and semi rigid joints should not be used. The structural analysis should either be elastic or by non-linear FEM. It should be noted that the plastic resistance may still be used for cross sections in Class 1 and 2.

The requirements on the material discussed above focus on the behaviour in tension. For instance, the ultimate to yield ratio is of importance for the ductility of a tension flange under steep stress gradient. In such a flange the plastic deformations before fracture may be confined

to a limited area that becomes smaller with decreasing ultimate to yield ratio. Some researchers have claimed that the ultimate to yield ratio also is important for avoiding premature local buckling in the plastic state. This is however a misconception that has been shown to be wrong in [2] on a theoretical basis and by experiments. Also experiments in [3] show that girders of steel with virtually no strain hardening show sufficient rotation capacity. It is clear that plastic hinge analysis is possible in certain cases but we need more studies before it can be allowed for general use with for instance S690 steel.

1.4 WELDED CONNECTIONS

One way of simplifying the welding of HSS is to use undermatched electrodes, which makes the welds more ductile and less prone to crack. Undermatched electrodes are not allowed in EN 1993-1-8 but they are allowed in EN 1993-1-12. The design of such undermatched welds should be based on the electrode strength rather than the base material strength. For fillet welds the parameter β_w is set to 1,0.

Undermatched butt welds were studied in [4] with 36 tests on different combinations of parent material and electrodes. The parent material was 30 mm thick S500 and S690 and the electrodes were from matching down to 80% of the parent material strength. The weld was an unsymmetrical double X-joint. All the tests showed a ductile behaviour with elongation at failure above 20% measured as A_5 over the weld. All tests showed higher strength than the electrode strength. The lowest strength noted was 90% of the base material strength in a combination with electrodes with 80% of the base material strength. This shows quite convincingly that it is safe to base the resistance on the electrode strength in case of undermatched electrodes.

Fillet welds were studied experimentally in [5]. The study reported 27 new tests and for evaluation of a design formula tests from literature was used as well, in total 420 tests. Only tests for which the strength of both base material and electrodes had been measured were used in the evaluation. The test data base comprised undermatched as well as overmatched electrodes. The results come out to be a modification of the design formula (4.1) of EN 1993-1-8 which reads

$$\sqrt{\sigma_\perp^2 + 2\tau_\perp^2 + 3\tau_{//}^2} \leq \frac{f_u + f_{eu}}{2\gamma_{M2}} \qquad (1)$$

The notations are the same as in EN 1993-1-8 with the additional symbol f_{eu} meaning the characteristic ultimate strength of the electrodes. This design criterion works for overmatched as well as undermatched electrodes and it gives a more uniform safety level for loading parallel and transverse to the weld than the criterion in EN 1993-1-8. A calibration of the safety level according to Annex D of EN 1990 resulted in $\gamma_{M2} = 1,25$.

This improved formula was however not introduced in EN 1993-1-12 as it concerned all kind of steels and that it need to be discussed in the scientific society before being accepted. It is here presented as a proposal for the revision of EN 1993-1-8. The step taken in EN 1993-1-12 of allowing undermatched electrodes and to use the electrode strength for such electrodes is clearly conservative and sufficient for the purpose of introducing HSS.

1.5 BOLTED CONNECTIONS

For bolted connections the main questions are the bearing resistance and the net section resistance. A study of bearing resistance and block shear resistance is presented in [6]. It reports results from tests on double shear lap joints of plates of steel grade S640. The conclusion was that the design rules of ENV 1993-1-1 were safe and that the rules for block shear were over-conservative. These rules have now been improved in EN 1993-1-8. Another conclusion was that the resistance of the connection may be taken as the sum of resistances of the bolts. In [7] and [8] results for bolted connection in steel grade S690 and S1100 are compared with S355. The resistance to bearing failure was always higher than the characteristic resistance according to EN 1993-1-8. The deformation at failure is smaller for the higher grades but the reduction is not large. The behaviour can still be seen as quite ductile. For instance connections with two bolts showed a deformation at failure of half a bolt diameter for S355 and 0,4 times the bolt diameter for S690 and S1100. This should be seen in relation to the strain at maximum load in tensile tests ε_u which was 15%, 7,5% and 3% for the three grades. This large difference is not reflected in the behaviour of the connection.

The resistance of net sections has been studied in [9] with bending tests and tensile tests with steel grade S690. The resistance turned out to be quite exactly $f_u A_{net}$ and the deformation at failure was as expected quite small. This will always happen if $f_u A_{net} < f_y A_{gross}$ also for low strength steel. The difference is that the low strength steel can tolerate larger holes than high strength steel. In EN 1993-1-1 the net section resistance is quite conservatively given as

$$N_{Rd,net} = \frac{0{,}9 f_u A_{net}}{\gamma_{M2}} \tag{2}$$

For HSS this resistance can come out unduly small and in EN 1993-1-12 there is an additional rule stating that the resistance need not be taken as smaller than

$$\frac{f_y A_{net}}{\gamma_{M0}} \tag{3}$$

1.6 LOCAL AND GLOBAL BUCKLING

There are numerous studies of the effect of buckling on structures of HSS. In general those studies show that HSS performs better than ordinary steel or at least not worse. That means that the normal design rules can be used as a conservative approach. The reason for HSS to behave better is a smaller influence of imperfections. For flexural buckling this is discussed for instance in [10] and [11]. The effect of bow imperfection is smaller because of the higher yield strength and the residual stresses are a smaller fraction of the yield strength. The latter observation was made in [11] and [12]. In EN 1993-1-1 this is reflected in the rules for flexural buckling by the use of higher buckling curves for S460. EN 1993-1-12 states that those curves may also be used for higher grades. A more rational way to account for the increased relative resistance would be to give a gradual increase of the buckling curves by modifying the imperfection parameter as suggested in [13].

For local buckling most of the results indicate a better performance of HSS. An exception is [12] reporting compression tests on welded boxes of steel up to S1100, which come out with more or less the same relative resistance as low strength steel.

2. DOES HIGH STRENGTH STEEL SAVE MONEY?

In order to study the economy of using HSS an estimate of prices is needed, which is a quite intricate question. The price of structural steel usually increases with the strength, which can be seen from Fig. 1. It shows relative prices for heavy plates from three leading European producers of HSS in which S235 has been chosen as reference. Fig. 1 also shows a trend curve, which follows the square root of the yield strength. Similar results have been shown for hot rolled strips. There is a substantial scatter in prices from time to time due to the market situation and the marketing strategy of the producer. Anyway, the trend curve in Fig. 1 will be used in this study as a reflection of probable prices.

If the strength can be fully utilised the cost of material will be lowered as the strength is increased, see Fig. 2. The cost of a structure depends however more on costs for fabrication and erection than on the price of the material. The savings in fabrication costs may be higher than the saving in material cost but they depend very much on the type of structure. For simplicity only the cost of material will be studied here.

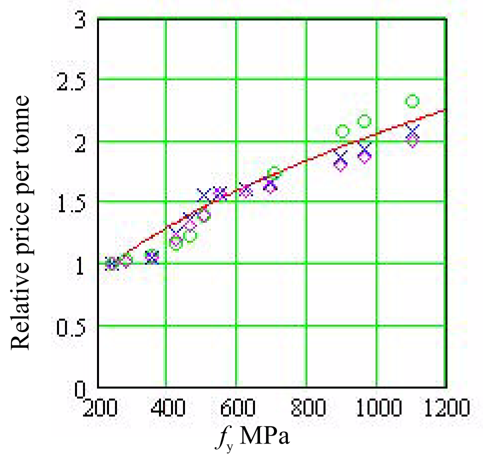


Fig. 1. Approximate price per tonne of hot rolled steel normalised with price of S235 as function of yield strength.

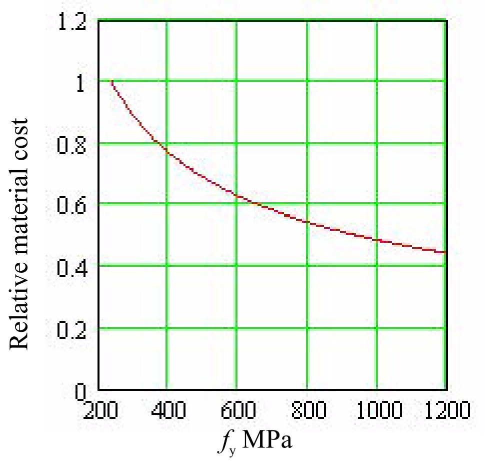


Fig. 2. Approximate material cost normalised with the cost of S235 assuming that the strength can be fully utilised.

When the resistance is governed by buckling the saving in material cost will be smaller than in Fig. 2 and it will depend on the type of buckling and the slenderness of the component. Fig. 3 shows the relative material cost for a column designed for flexural buckling according to curve a. It can be seen that you save money on stocky columns if the strength is increased (l/i=40). For l/i = 60 where is a minimum of cost at f_y = 600 MPa. This is compatible with the fact that HSS is used in columns in high rise buildings. Results for plate buckling are shown in Fig. 4 where the slenderness is represented by width over thickness. It is clear that increasing the yield strength always reduces the material cost but for slender plates the saving is quite small. In order to get a substantial saving the plate slenderness has to be kept low.

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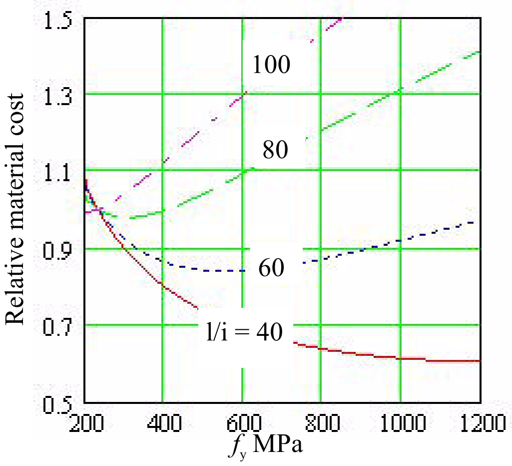


Fig. 3. Relative material cost for column subject to flexural buckling (curve a) as function of yield strength f_y. Reference cost is for steel S235.

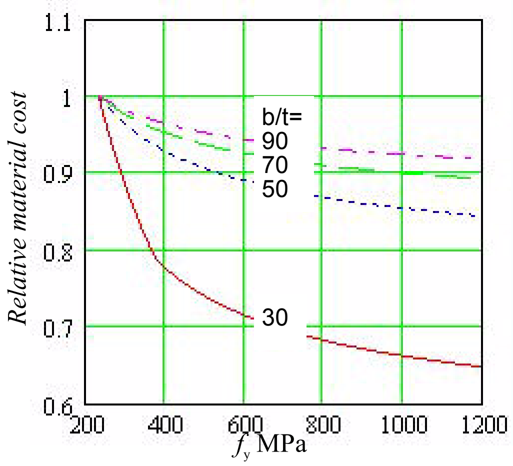


Fig. 4. Relative cost for a plate supported along all edges and subject to uniform compression as function of yield strength f_y. Reference cost is for steel S235.

3. APPLICATIONS OF HIGH STRENGTH STEEL IN CONSTRUCTION

3.1 HYBRID GIRDER BRIDGE

In Sweden it is common practice to mix different steel grades within a cross section, with the strongest in the flanges and the weaker grade in the web. These cross-sections of mixed type are known as “hybrid girders”, and are regulated in the Swedish Bridge Code. This document states that the strength of the flanges may not exceed that of the webs by more than 50%. Furthermore, when the web is reduced with respect to buckling of normal stresses, the effective web thickness is calculated based on the strength of the flange, not the web itself.



Fig. 5. Bridge Z1207 in Mittådalen.

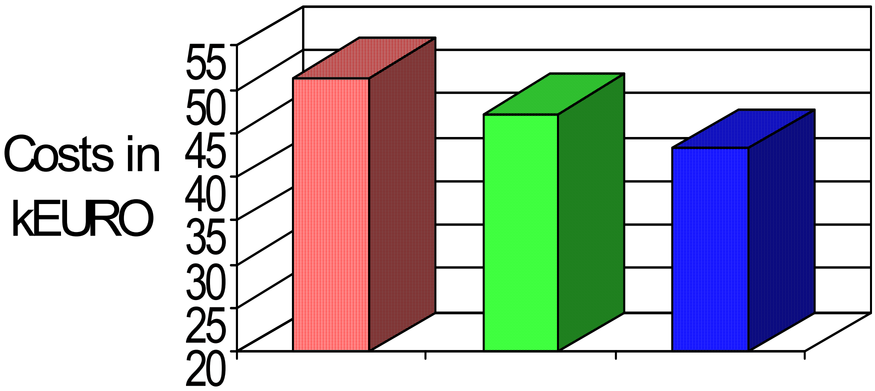


Fig. 6. Cost savings yielded by three mixes of material; S355 all over the cross section; S460 in only the bottom flange and to the right; S460 in the web and top flange, mixed with S690 in the bottom flange.

In 1995, a bridge with S690 (WELDOX 700) in the lower flange was erected in Mittådalen. In Fig. 5, prefabricated concrete deck elements are mounted on the steel girders. In this case, the steel contractors had three alternatives to evaluate. It was calculated that the solution adopting S690 in the lower flange and S460 in the web and upper flange proved the least costly choice. Using S690 in the upper flange is less cost-effective, as it is often submitted to buckling/lateral-torsional buckling during the casting cycle.

In the case of continuous bridges, the same applies to mid span regions, whereas the opposite is true for the support area, i.e. tensile force in the upper and compression in the lower flange. The latter is determined mainly by lateral-torsional buckling, implying that a transition from e.g. S460 to S690 necessitates a reduction in the width/thickness ratio of the flange to reach the fully elastic moment. This slenderer flange in turn requires a substantial reduction in the distance between the cross bracings, in order to avoid lateral-torsional buckling of the lower flange. Contrarily, the upper flange is subject to tensile forces at supports; however, using steel grade S690 would cause excessive reinforcement stresses as well as cracks in the concrete deck. Furthermore, fatigue problems may occur due to the shear connectors welded to the top flange.

One reason for the limited use of this concept for simply supported bridges and the midspan girders of continuous bridges is the longer delivery time for QT-steels in comparison to TM steels. In many cases, considering the narrow time margins where sometimes both the order of materials and earthworks are carried out without owner-approved documentation, increased delivery times is not an option. An increased use of S690 in bridges would thus require an improved delivery stand-by time from the steel mills. At a glance, the short time spans available today for executing the projects might appear to be the result of bad planning by the Swedish National Road Administration, but is in reality intimately connected to insecurity regarding funding and prioritizing from the Government.

3.2 NESENBACHTALBRÜCKE, GERMANY

Nesenbachtalbrücke close to Stuttgart is a 572 m long composite bridge with span lengths of 35,10 to 89,50 m. The bridge was built during 1998 – 2000, and designed by Schlaich, Bergermann und Partner. The architectural requirement for slender filigree structures can be achieved by hollow section lattice structure with small construction depth and diameter of members. The columns are integrated in the shape of trees with trunks and branches with high slenderness ratios.

This concept leads to peak stresses in particular areas, e.g. at the connections between beams and columns where the slender dimensions of members could be maintained by the use of S690QL1.



Fig. 7. The Nesenbachtalbrücke.

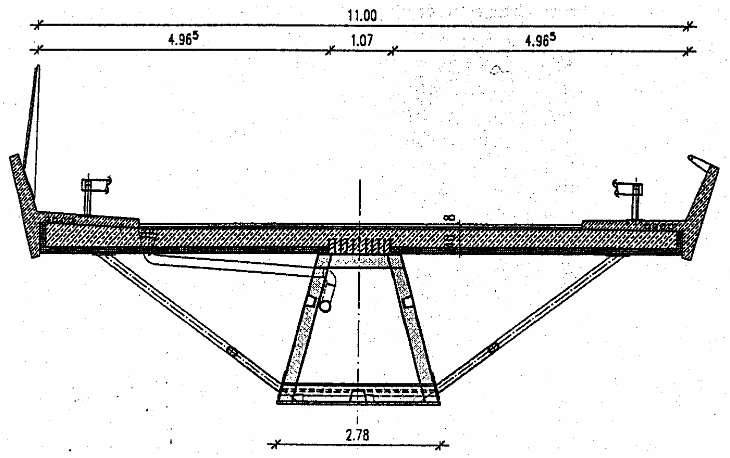


Fig. 8. Cross section of the Nesenbachtalbrücke.

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3.3 SONY CENTER, BERLIN

Various storeys of a hotel building at the Sony Center in Berlin are suspended from a roof truss to protect an old masonry building with the "Kaisersaal" integrated in the hotel building from overloads. The truss structure composed of components with a solid rectangular shape was made of S 460 and S 690. High strength steels were used to keep the dimensions of the cross sections small. The verification to avoid brittle fracture at low temperatures was performed by calculations according to EN 1993-1-10 assisted by testing. The structural detailing and the dimensions of the "large scale test specimens" are indicated in the following figures.

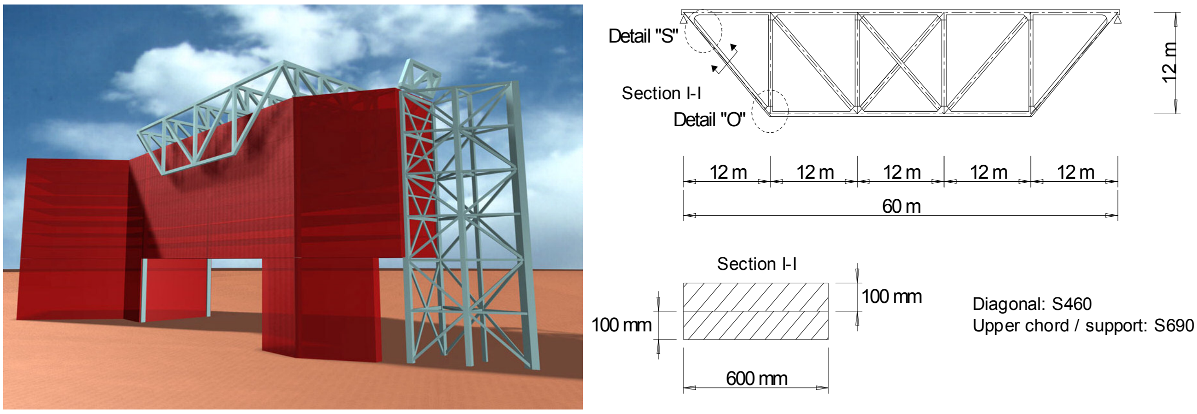


Fig. 9. Truss structure in theSony Center.

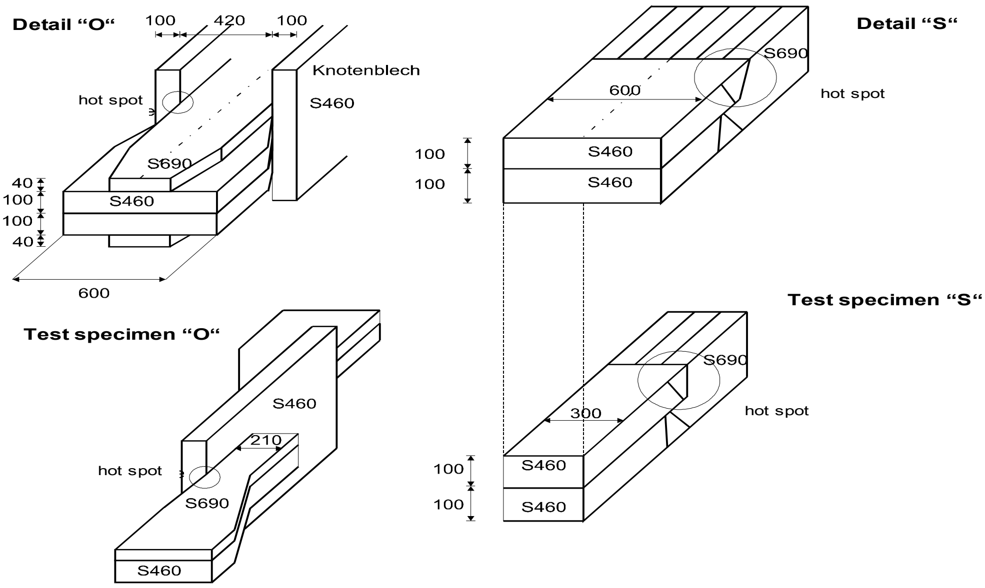


Fig.10. Structural detailing and large scale specimens.

3.4 MILITARY BRIDGE IN HSS

Karlskronavarvet AB, a shipyard producing mainly navy ships, has developed a new military bridge, Fast Bridge 48, for the Swedish army. The specification for the bridge stated that it should be designed for MLC 70. It shall resist a 65 t tank for 1000 crossings, the clear span should be up to 46 m and the bridge should be transported by standard army lorries and be erected in 75 minutes by six men. The selected solution was a truss bridge of which one element is shown in Fig 5 [15]. The bridge is erected as follows. First a launching beam is cantilevered over the river or gap, which

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allows one soldier to cross and arrange a support for the launching beam. The truss elements are lifted on to the launching beam, connected to each other with fast couplings and pushed out on the launching beam. The bridge can be retrieved from either side. At a later stage the system has been developed further to make it possible to build continuous bridges, which is shown in Fig. 6.

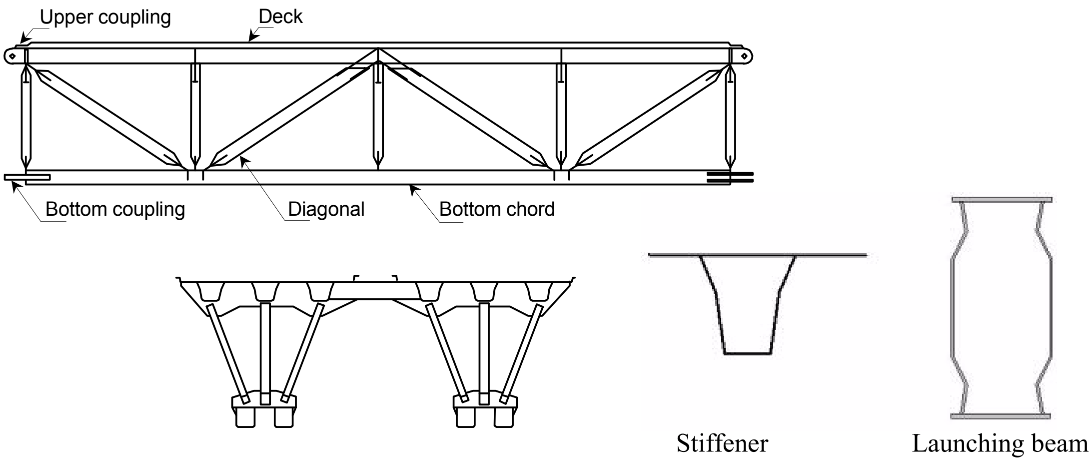


Fig. 11.Truss element for Fast Bridge 48 developed by Karlskronavarvet AB, Sweden.

Some structural details may be of interest in this context. In order to keep the weight as low as possible HSS was chosen. Most of the steel in the bridge is S1100, which is cold-formed and welded. The deck is 5 mm thick and it is stiffened by six longitudinal trapezoidal stiffeners also of 5 mm material. The stiffeners and the launching beam have slight longitudinal folds on the webs, see Fig. 5, which reduces the slenderness for local buckling. The working stresses are in the magnitude of 700 MPa and it is therefore obvious that the bridge will sag substantially. There are however no restriction of the sag in the specification and the bridge actually sags 0,65 m (span/70) under full working load but this does not harm its function. Another problem with the high stress level is fatigue. Although the requirement was only 1000 cycles the fatigue resistance was a concern. Especially the detailing of the connection between coupling and chords was critical and it was also tested in fatigue together with some other details.

This example is quite different from normal structures in the construction area. The military bridge is a product that will be manufactured in numbers that justifies an investment in product development. It shows also that there is a need to use new structural solutions in order to make full use of the HSS.



Fig. 12. Fast bridge used as a continuous bridge loaded by a tank.

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4. CONCLUSIONS

High strength steel is continuously winning market shares in the construction industry, although the pace could be higher. HSS can bring along significant savings, but buckling, fatigue or deflection criteria often set the limit for the steel grade being used. Sometimes these limits can be pushed towards higher grades, by using innovative structural solutions. Another obstacle has been the lacking of rules for grades higher than S460 in Eurocode 3, but a working group within CEN/TC250/SC3 has recently finished a Part 1-12 of Eurocode 3 that extends its scope to S700. This will indeed make it easier for the designers and the steel contractors to choose HSS in the future.

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